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SYNOPSIS Compacted clay liners and covers are widely used in waste containment units. Case histories in three categories are presented: (1) case histories illustrating compaction, construction, and quality assurance difficulties; (2) case histories involving field hydraulic conductivity testing of large-scale test pads; and (3) case histories involving final cover systems. The case histories illustrate that: (1) compaction criteria should be chosen carefully and with consideration given to how the compaction will be controlled in the field; (2) regulatory roadblocks may defeat sound technical approaches in terms of developing compaction criteria; (3) one can follow ASTM procedures and still get into difficulty if sample preparation procedures are not given special attention; (4) data on field performance of test pads provides valuable insight concerning the relationship between hydraulic conductivity and field performance; and (5) problems with differential settlement, desiccation, and freeze-thaw make use of compacted clay liners a challenge in final cover systems -- geosynthetic clay liners offer an attractive alternative.

INTRODUCTION

This paper describes case histories concerning field performance of low-permeability, compacted soil liners for waste containment applications. Compacted soil liners are used as hydraulic barriers in liner systems (Fig. 1) and final cover systems (Fig. 2) for new waste containment units as well as for site remediation projects. Compacted soil liners of one type or another are a required component by regulation for nearly all modern waste disposal facilities.

In this paper, case histories are reviewed that illustrate problems encountered in construction of soil liners, construction quality assurance of soil liners, hydraulic conductivity of test pads, and performance of compacted clay used in final cover systems. Before the case histories are presented, a brief review of fundamental principles is presented primarily for the benefit of the reader who is inexperienced with compacted soil liners.

BACKGROUND ON SOIL LINERS

Compacted soil liners are constructed primarily from natural soil materials, although bentonite-soil blends are also used. Soil liners are constructed in lifts that typically have a maximum thickness after compaction of 150 mm (6 in.). On side slopes, the lifts can be horizontal or parallel to the slope.

Compaction Objectives

The objective of compaction is 1) to densify the soil (hydraulic conductivity is proportional to the inverse of the porosity cubed), and 2) to remold soil aggregates ("clods") of soil into a homogeneous mass that is free of large, continuous, interclod voids. If these objectives are accomplished with suitable soil materials, low hydraulic conductivity ($\leq 1 \times 10^{-7}$ cm/s) will result.

Experience has shown that the water content of the soil, method of compaction, and compactive effort have a major influence on the hydraulic conductivity of compacted soil liners. Laboratory studies have demonstrated that low hydraulic conductivity is easiest to achieve when the soil is compacted wet of optimum water content with a high level of kneading-type compactive energy (Mitchell, Hooper, and Campanella, 1965). The soil must be sufficiently wet so that, upon compaction, clods of clayey soil will mold together, eliminating large inter-clod pores. The water content of the soil at the time of compaction is the single most important factor that affects hydraulic conductivity of compacted soil liners. Soft, wet pieces of clay are much easier to remold into a homogenous, low hydraulic conductivity mass than dry, hard pieces. Kneading the soil during compaction with a high level of compactive energy also helps to remold clods and to eliminate large pore spaces.

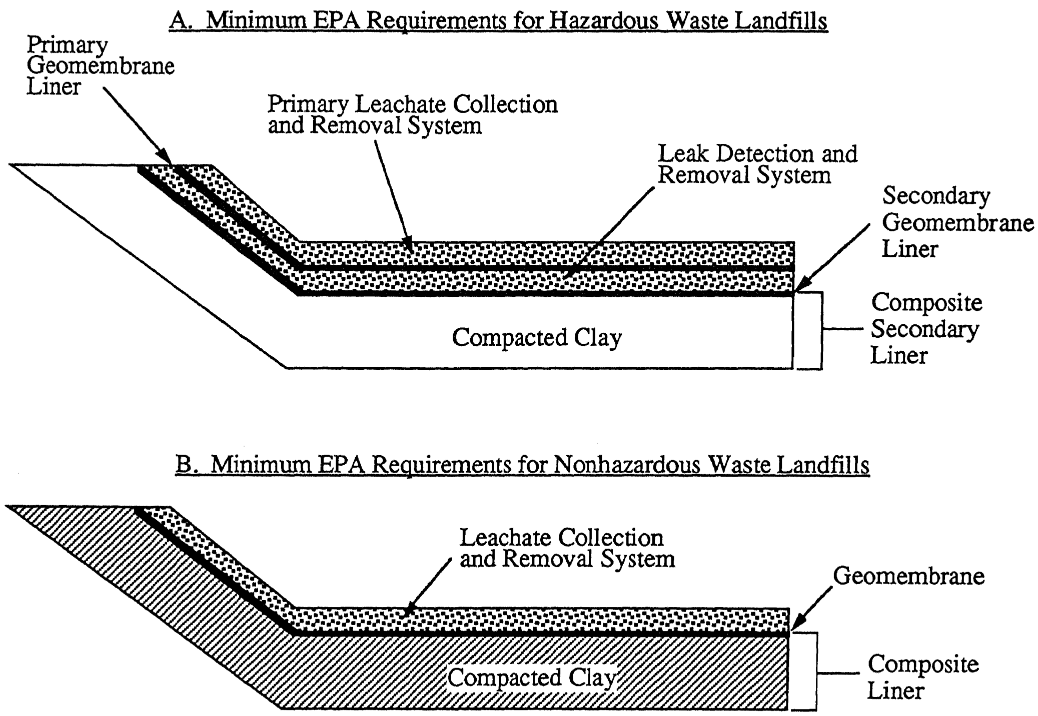
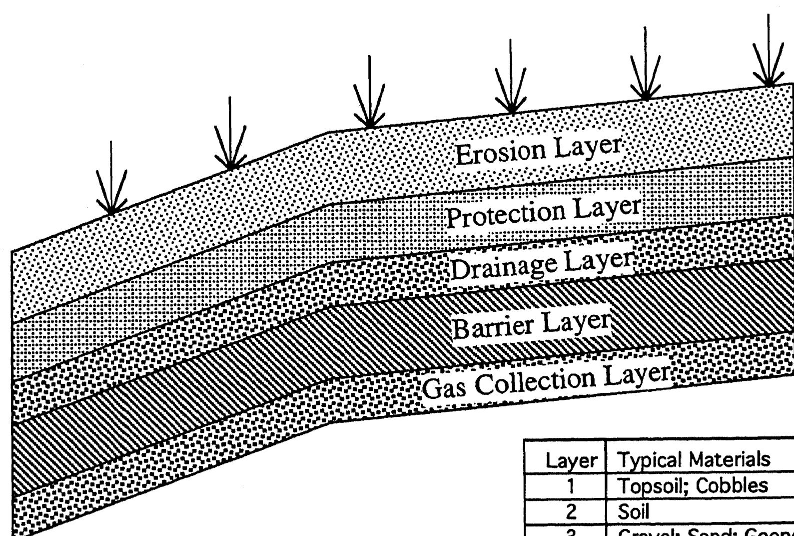


Figure 1. Typical Minimum Profiles for Liner Systems in Modern Landfills.



Layer	Typical Materials
1	Topsoil; Cobbles
2	Soil
3	Gravel; Sand; Geonet
4	Compacted Clay; Geomembrane; Geosynthetic Clay Liner
5	Gravel; Sand; Geonet; Geotextile

Figure 2. Components of Final Cover Systems.

Studies have also demonstrated that lifts of soil must be effectively bonded together to minimize highly permeable zones at lift interfaces. If permeable inter-lift zones are eliminated, hydraulic connection between "defects" in each lift is destroyed and a low overall hydraulic conductivity is achieved.

Materials

The recommended requirements for soil liner materials that must have a hydraulic conductivity $\leq 1x10^{-7}$ cm/s are as follows:

Percentage of Fines:	≥ 30 to 50%
Plasticity Index:	≥ 7% to 10%
Percentage of Gravel:	≤ 20% to 50%
Maximum Particle Size:	25 to 50 mm

Percentage of fines is defined as the percent by dry weight passing the U.S. No. 200 sieve, which has openings of 0.075 mm. Plasticity index is determined by ASTM D-4318. Percentage of gravel is defined as the percent by dry weight retained on a No. 4 sieve (4.76 mm openings). Local experience may dictate more stringent requirements, and for some soils, more restrictive criteria may be appropriate. However, if the criteria tabulated above are not met, it is unlikely that a natural soil liner material will be suitable without additives such as bentonite.

If suitable materials are unavailable locally, local soils can be blended with commercial clays, e.g., bentonite, to achieve low hydraulic conductivity. A relatively small amount of sodium bentonite can lower hydraulic conductivity as much as several orders of magnitude. There are no specific material requirements for soils that are to be blended with bentonite. Any material can be made to have a hydraulic conductivity $\leq 1 \times 10^{-9}$ m/s; the only issue is how much bentonite is needed. Soils with a broad range of grain sizes usually require a relatively small amount of bentonite (≤ 6% on a dry-weight basis). Uniform-sized soils, such as a dune sand, usually require more bentonite (up to 10 to 15%). Sometimes materials are blended to provide a material with a broad range of grain sizes and to minimize bentonite content. For instance, on one project the designer recommended blending waste fines with a coarse sand to minimize the amount of bentonite required.

Construction Procedures

Some liner materials need to be processed to break down clods of soil (Benson and Daniel, 1990), to sieve out stones and rocks, to moisten the soil, or to incorporate additives. Clods of soil can be broken down with tilling equipment. Stones can be sieved out of the soil with large vibratory sieves, mechanized "rock pickers" passed over a loose lift of soil, or by laborers who remove oversized material by hand. Road reclaimers can process soil in a loose lift and breakdown hard chunks and crush stones or large clods (Daniel, 1991a).

If the soil must be wetted or dried more than 2-3 percentage points in water content, the soil should be processed by spreading it in a loose lift about 300 mm (1 ft) thick. If the soil is to be dried, the soil should be periodically tilled to promote uniform drying. The soil can be moistened by uniformly applying water to the surface of a loose lift and mixing. It is essential that time be allowed for the soil to wet or dry uniformly. At least 1-3 days is usually needed for adequate hydration or dehydration. Frozen soil should never be used to construct a soil liner.

Additives such as bentonite can be introduced in two ways. One technique is to mix soil and additive in a pugmill. Water can also be added in a pugmill either concurrently with bentonite or in a separate processing step. Alternatively, the soil can be spread in a loose lift 150-300 mm (6 to 12 in.) thick, the additive spread over the surface, and rototillers used to mix the materials. Several passes of the tiller over a given spot are usually needed. Water can be added in the tiller during mixing or later, after mixing is complete. The pugmill is more reliable in providing thorough, controlled mixing.

It is crucial that each lift of a soil liner be effectively bonded to the overlying and underlying lifts. The surface of a previously-compacted lift must be rough rather than smooth. Many contractors like to smooth roll the completed lift of soil with a smooth steel-drummed roller. The smooth-rolled surface is desirable to promote runoff from rainstorms (a rough surface holds water in tiny puddles -- it may take several days to dry out the soil so that construction can resume) and to provide a hard skin that minimizes desiccation. However, if the next lift is placed on a hard, smooth surface, a distinct lift interface will develop, and the interface provides hydraulic connection between permeable zones in adjacent lifts. On the other hand, if the surface is rough, the new and old lifts blend into one another. Discs are used to scarify the surface of a previously-compacted lift to a depth of about 25 mm (1 in.).

Soil is placed in a loose lift that is no thicker than about 230 mm (9 in.). If grade stakes are used to gauge thickness, the stakes must be removed and the hole left by the stakes sealed. Other techniques that avoid use of stakes, e.g., use of lasers, are preferable for control of elevations. After the soil is placed, a small amount of water may be added to offset evaporative losses, and the soil may be tilled one last time prior to compaction.

Heavy, footed compactors with large feet that fully penetrate a loose lift of soil are ideal. Rollers with feet that fully penetrate a loose lift of soil pack the base of a new lift into the surface of the previously-compacted lift, which helps to bond lifts together. The long feet also help to break down and remold clods of soil over the full thickness of a lift. Recommended compactor specifications include a minimum mass of 18,000 kg (40,000 lbs) and minimum foot length of 180 to 230 mm (7 to 9 in.), but the foot should have a length no smaller than the thickness of a loose lift (counting the depth of scarification of the previously compacted lift. It is also recommended that the compactor make at least 5 passes over a given area -- even more passes will often be needed. A "pass" is defined as one pass of a self-propelled compactor, not just an axle, over a given area.

Statically operated compactors are usually preferred over vibratory compactors for soil liners. The weight of the compactor must be compatible with the soil: relatively dry soils with firm clods require a very heavy compactor whereas relatively wet soils with soft clods require a roller that is not so heavy that it becomes bogged down in the soil. Also, it is sometimes desirable to compact the lift with two compactors. A heavy roller with fully penetrating feet compacts the soil initially. If this roller leaves loose material in the upper part of the lift, a roller with short feet (*pad foot* roller), rubber-tired equipment, or a smooth steel-drum roller can be used to compact the upper part of a lift.

Soil-bentonite liners can often be compacted with rubber-tired or smooth-drum rollers. Soil-bentonite mixtures often do not develop clods, and densification of the soil is often the primary objective with soil-bentonite liners. However, rollers with fully-penetrating feet may be effective in bonding soil-bentonite lifts.

Earthwork contractors experienced in constructing dense structural fills expect a footed roller to "walk out" of the lift after repeated passes of the compactor. The compactor compacts the soil from the bottom up, and the feet penetrate the lift less and less with repeated passes. However, soil liners are placed and compacted at much larger water contents than structural fills. Footed rollers often do not walk out of wet fill materials. There is no reason for alarm if the roller does not walk out; tests on the soil are performed on the previously-compacted lift. The roller remolds the upper lift and compacts the underlying lift. It is important to educate contractors to understand that the main requirement for soil liner construction is remolding of the soil and not just compaction (densification) of the soil.

After compaction of a lift, the soil must be protected from desiccation and freezing. Desiccation can cause cracking of the clay (Chamberlain and Gow, 1979; Othman and Benson, 1991; and Kim and Daniel, 1992). Freeze-thaw causes compacted clay to crack and increases hydraulic conductivity. Desiccation can be minimized in several ways: the lift can be temporarily covered with a sheet of plastic (but one must be careful that the plastic does not heat excessively and thereby dry the clay), the surface can be smooth-rolled to form a relatively impermeable layer at the surface, or the soil can be periodically moistened. The compacted lift can be protected from frost damage by avoiding construction in freezing weather or by temporarily covering the lift with an insulating layer. The protective measures discussed in this section apply to each lift as well as to the completed liner.

Quality Control and Quality Assurance

Critical facets of liner technology are construction quality assurance (CQA) and construction quality control (CQC). Herein, per definitions used by the U.S. Environmental Protection Agency, CQC refers to observations and tests performed by the contractor to ensure quality construction. CQA refers to observations and tests performed by an organization that is independent of the contractor to verify quality.

For soil liners, CQC and CQA tests fall into three categories: 1) tests on subgrade; 2) tests and observations to verify that the soil liner materials are adequate, and 3) tests and observations to verify that the compaction process is adequate. Tables 1 and 2 summarize some of the author's recommendations concerning tests and observations on materials and construction procedures.

Table 1. Recommended CQA/CQC Tests To Verify that Proper Materials Are Used for Construction of the Low-Permeability Compacted Soil Liner.

Parameter	Test Method	Minimum Testing Frequency	Notes
Percent Fines	ASTM D-1140	1 per 1,000 yd^3	1,2,7
Percent Gravel	ASTM D-422	1 per 1,000 yd^3	2,3,7
Liquid & Plastic Limits	ASTM D-4318	1 per 1,000 yd^3	2,7
Percent Bentonite	See Note 4	1 per 1,000 yd^3	2,7
Compaction Curve	As Specified	1 per 5,000 yd^3	7
Water Content (Rapid)	ASTM D-4643	1 per 200 yd^3	2,5,7
Water Content (Oven Drying)	ASTM D-2216	1 per 2,000 yd^3	6,7
Material Excavation	Observation	Continuous	

Notes:
1. Percent fines is defined as percent passing the No. 200 sieve.
2. In addition, at least one test should be performed each day that soil is excavated or placed, and additional tests should be performed on any suspect material observed by CQA personnel.
3. Percent gravel is defined as percent retained on the No. 4 sieve.
4. There is no standard test method. The methylene blue test or alternative technique is recommended. This test is only applicable to soil-bentonite liners.
5. This is a microwave oven drying method. Other methods may be used, if more appropriate. Any method used besides direct drying via ASTM D-2216 should be calibrated against ASTM D-2216 for the on-site soils.
6. Microwave oven drying and other rapid measurement methods may involve systematic errors. Conventional oven drying (ASTM D-2216) is recommended on every 10-th sample taken for rapid measurement. The intent is to document any systematic error in rapid water content measurement.
7. 1000 yd^3 = 760 m^3.

Table 2. Recommended CQA/CQC Tests To Verify that Soil Has Been Compacted Properly.

Parameter	Test Method	Minimum Testing Frequency	Notes
Water Content (Rapid)	ASTM D-3017, ASTM D-4643, ASTM D-4944, ASTM D-4959	5/acre/lift	1,2,6
Water Content (Oven Drying)	ASTM D-2216	0.5/acre/lift	3,6
Density (Nuclear)	ASTM D-2922	5/acre/lift	2,4,6
Density (Non-Nuclear)	ASTM D-1556, ASTM D-2167	0.25/acre/lift	5,6
Number of Passes	Observation	1/acre/lift	2,6
Construction Oversight	Observation	Continuous	

Notes:
1. ASTM D-3017 is a nuclear method, D-4643 is microwave oven drying, D4944 is rapid analysis with a calcium carbide gas pressure tester, and D-4959 is rapid direct heating. Direct water content determination (ASTM D-2216) is the standard against which nuclear, microwave, or other methods of measurement are calibrated for on-site soils.
2. In addition, at least one test should be performed each day soil is compacted and additional tests should be performed in areas for which CQA personnel have reason to suspect inadequate compaction.
3. Every tenth sample tested with ASTM D-3017, D-4643, D-4944, or D-4959 should be also tested by direct oven drying (ASTM D-2216) to aid in identifying any significant, systematic calibration errors.
4. ASTM D-2922 is a nuclear method. This method, if used, should be calibrated against the sand cone (ASTM D-1556) or rubber balloon (ASTM D-2167) for on-site soils.
5. Every twentieth sample tested with D-2937 should also be tested (as close as possible to the same test location) with the sand cone (ASTM D-1556) or rubber balloon ASTM D-2167) to aid in identifying any systematic calibration errors with D-2937.
6. 1 acre = 0.4 ha.

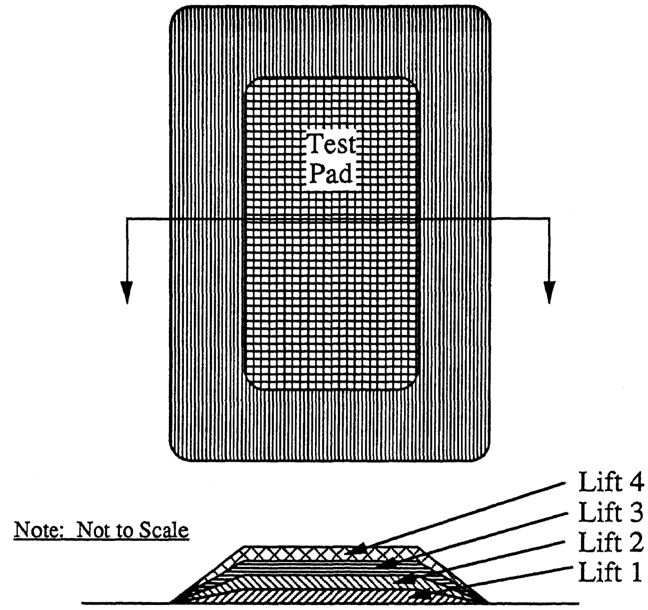


Figure 3. Test Pad

Test Pads

The construction of a test pad (Fig. 3) prior to building a full-sized liner has many advantages. By constructing a test pad, one can experiment with compaction water content, construction equipment, number of passes of the equipment, lift thickness, etc. Most importantly, though, one can conduct extensive testing, including quality control testing and in-situ hydraulic conductivity testing, on the test pad to verify performance criteria and the effectiveness of proposed CQA procedures.

It is usually recommended that the test pad have a width of at least 3 construction vehicles and an equal or greater length. The pad should ideally be the same thickness as the full-sized liner, but the trial pad may be thinner than the full-sized liner. (The full-thickness liner should perform at least as well as, and probably better than, a thinner test pad). The in-situ hydraulic conductivity may be determined in many ways. The sealed double-ring infiltrometer (SDRI) is usually the best large-scale test (Daniel, 1989a; Sai and Anderson, 1990; and ASTM D-5093), although the Boutwell test (Daniel, 1989, and references therein) is enjoying increased popularity due to its ease of operation and relatively short testing times.

One problem with in situ hydraulic conductivity tests on test pads is that the test pad is subjected to essentially zero overburden stress. Hydraulic conductivity decreases with increasing compressive stress. The hydraulic conductivity measured in situ on a test pad may be corrected for the effects of overburden stress based on results of laboratory hydraulic conductivity tests performed over a range in compressive stress.

CASE HISTORIES OF CONSTRUCTION PROBLEMS

Site in Southeastern U.S.

This project involved construction of a composite geomembrane/clay cap over several contaminated sites. The owner retained an engineering company to investigate potential borrow soils. It was understood that the soils had to have a hydraulic conductivity less than or equal to 10^{-7} cm/s, but due to a misunderstanding, there was confusion over whether the requirement meant that the hydraulic conductivity had to be less than or equal to 1×10^{-7} cm/s or just within the 10^{-7} cm/s range. It was finally determined that the hydraulic conductivity had to be less than 1×10^{-7} cm/s.

Samples of potential borrow soils were obtained and shipped to a geotechnical laboratory for testing. The laboratory personnel took the soil, air dried the material, sieved the soil through a No. 4 sieve, discarded all material retained on the No. 4 sieve, and prepared test specimens for permeation. The specimens were prepared as follows. A separate compaction test was performed (ASTM D698), and optimum water content (w_{opt}) and maximum dry unit weight ($\gamma_{d,max}$) were determined. It was anticipated that the construction specifications would require that the soils be compacted to a minimum dry unit weight equal to 95% of the value from ASTM D698. Accordingly, batches of soil were mixed to several different water contents. The amount of soil needed to fill a compaction mold of known volume at a specified dry density ($0.95 \cdot \gamma_{d,max}$) and water content was determined and weighed. The soil was compacted into the mold using a tamping rod until the pre-determined amount precisely filled the mold. Thus, the desired dry unit weight was achieved for several different water contents. The specimens were extruded from the compaction molds, set up in a flexible-wall permeameter, back-pressure saturated, and permeated. The tests were performed shortly before publication of ASTM D5084 for hydraulic conductivity testing of soils having low hydraulic conductivity. The laboratory personnel followed the instructions of the manufacturer in calculating hydraulic conductivity.

Unfortunately, the hydraulic conductivities that were measured on the laboratory-compacted materials were generally greater than 1×10^{-7} cm/s, but less than 1×10^{-6} cm/s. Originally, due to the misunderstanding over the required hydraulic conductivity, it was thought that this range of values was acceptable, but after further investigation, it was found that these values were too high.

It was at this point that the author became involved. The soils in question had relatively favorable properties; about 70% of the material passed the No. 200 sieve, the liquid limit averaged about 55%, and the plasticity index was about 25%. Based on the author's experience, he had little doubt that this type of material could be compacted to produce a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s, and yet the testing laboratory's results did not indicate that this was the case.

The author visited the testing laboratory and carefully examined their soil preparation procedures and testing procedures. Because there is no guidance from ASTM on how to prepare a test specimen for hydraulic conductivity evaluation, and at the time there was not even an ASTM standard for hydraulic conductivity testing, it would not be surprising to find problems from time to time. In this case, several problems were identified, the most important of which were:

- When technicians air dried and then sieved the soil, all materials retained on the No. 4 sieve were discarded. The materials retained on the No. 4 sieve, as it turned out, included some pieces of chert but many dry clods of clayey soil. No attempt was made to break up the dry clods of cohesive material. In effect, the technicians were sieving out the most active of the clay fraction, i.e., the very material most responsible for low hydraulic conductivity!

- When the soils were compacted, a pre-determined amount of soil was compacted into a mold. This is a very common practice, but the practice suffers from the problem that there is no control over compactive energy. Mitchell et al. (1965) have shown that one can change the hydraulic conductivity of a soil by 100-fold without changing the dry unit weight simply by modifying the energy of compaction delivered to the soil.

- The laboratory personnel followed the equipment manufacturer's recommendations on computation of hydraulic conductivity. The tests involved a falling head. However, while the water level in the influent reservoir fell, the water level in the effluent reservoir rose. This is termed a "falling headwater-rising tailwater test." The manufacturer's manual recommended an equation for computation of hydraulic conductivity that is appropriate for a test with a falling headwater and constant tailwater level. This equation gives a hydraulic conductivity that is too large by a factor of 2 (Daniel, 1989b).

The owner of the site decided to have the hydraulic conductivity tests repeated. The author and Mr. Craig H. Benson, then a graduate student at the University of Texas, conducted the tests. It was recognized that the problem of producing test specimens using known and reasonable compactive energies was best solved by using well recognized compaction techniques. Three compactive energies were used to span a range of reasonable compactive effort:

1. Modified Proctor (ASTM D-1557) was used to simulate a reasonable upper limit on compactive energy.

2. Standard Proctor (ASTM D-698) was used to represent a modest level of compactive energy.

3. A procedure that was termed "reduced Proctor" was developed to simulate a reasonable lower bound of compactive energy. The reduced Proctor procedure was the same as ASTM D-698, except that only 15 blows of the ram per lift were used rather than the usual 25.

Samples of 3 different types of materials from two borrow areas were compacted and permeated at the University of Texas. It was found that the soils could be compacted to produce a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s. However, the conventional type of compaction specification in which the soil is compacted to a specified minimum dry unit weight over a specified range of water content did not work well in defining water content-density values that produced satisfactory hydraulic conductivities. This led to the development of the procedure outlined by Daniel and Benson (1990) and summarized in Fig. 4.

An Acceptable Zone of water content-dry unit weight was developed based on the test results and was implemented when construction began. The methodology seemed to work well, and no major problems were encountered during construction.

The lessons learned from this project may be summarized as follows:

1. There should be a clear and unambiguous understanding of the hydraulic conductivity requirements as the beginning of a project.

2. There is no standard procedure for processing soil materials and compacting test specimens for hydraulic conductivity testing. The methods used can have an important effect on the outcome of the tests. Engineers should track these details carefully.

3. Experience showed that an Acceptable Zone of water content-dry unit weight values, as indicated in Fig. 4, could be developed and successfully implemented.

Site in Midwest

An owner of a large contaminated site in the midwest was required to construct a clay cover over the contaminated area. The rules of the state regulatory agency require that soil liner materials be compacted at a water content that is 0 to 4% wet of optimum and to a minimum dry unit weight of either 95% of the maximum value from ASTM D-698 or 90% of the maximum value from ASTM D-1557. The owner knew of the acceptable zone concept just described and desired to apply it, despite the rules of the state regulatory agency that required a more traditional approach. It was hoped that an exception would be granted, provided a compelling body of test data supported an alternative acceptable zone. The author was involved in helping to define an appropriate acceptable zone.

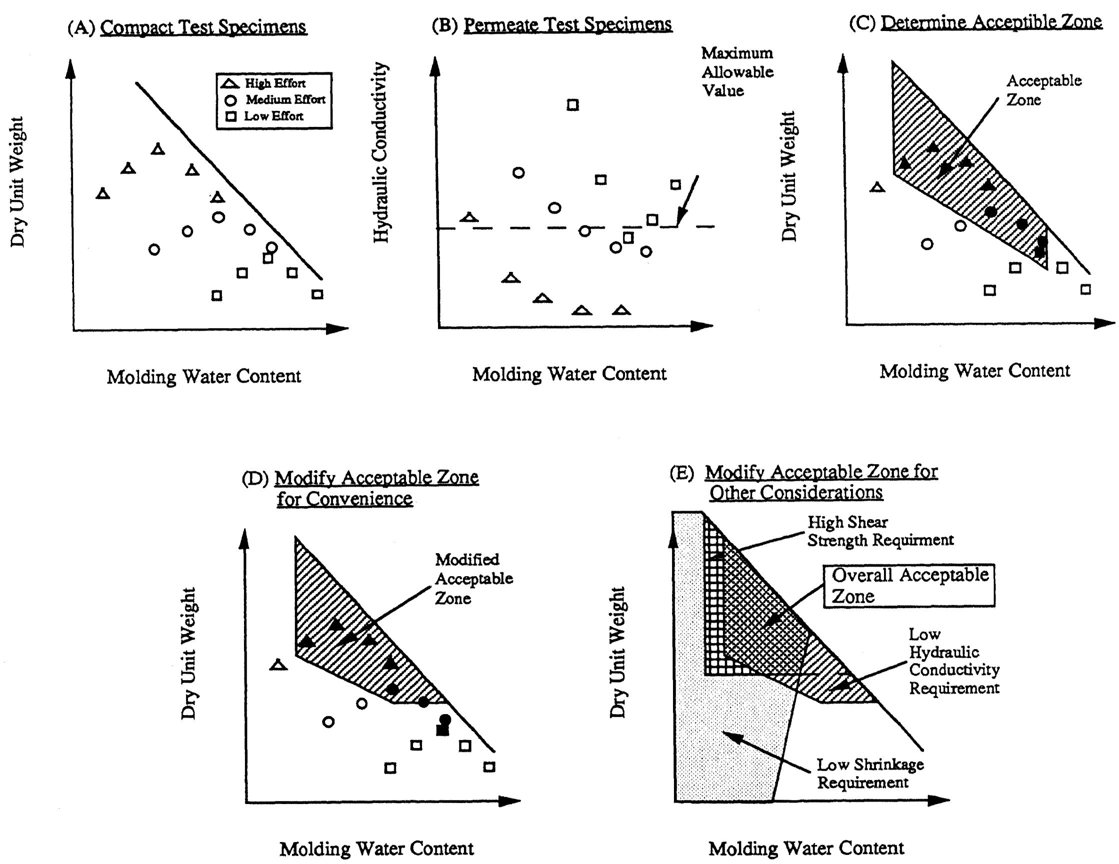


Figure 4. Procedure for Defining Water Content - Dry Unit Weight Criterion (after Daniel and Benson, 1990).

Samples of material were obtained from the field and shipped to a local testing laboratory. The soils were compacted using a similar procedure to the one dicussed in the previous case history, i.e., a predetermined amount of soil was packed into a mold to produce a known dry unit weight. The author was not involved in determining the test procedures, although, as discussed later, he did have some comments, and additional tests were performed to address these comments. The highly plastic soil was found to be extremely good in terms of producing low hydraulic conductivity over a broad range in water content (Fig. 5).

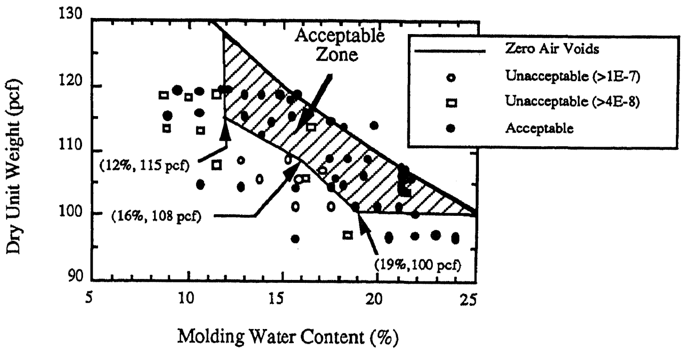


Figure 5. Results of Hydraulic Conductivity Tests.

When the testing laboratory completed the tests, the author reviewed the results. Several questions were raised:

1. The laboratory sieved the soils through a No. 4 sieve. Although clods of clay were crumbled to pass through the sieve, one wonders about processing materials in this way when the soil will not be sieved in a similar manner in the field. Additional tests were performed without sieving, and little difference was noted. For one typical compaction condition, the average hydraulic conductivity of the sieved material was 4.4×10^{-9} cm/s while that of the unsieved material was 5.5×10^{-9} cm/s. The difference was small.

2. The method of compaction employed by the testing laboratory did not involve control of the compactive energy. Three test specimens were prepared by standard Proctor compaction (ASTM D-698), and three were compacted with the laboratory's technique to the same average dry unit weight. The average hydraulic conductivities were 4.4×10^{-9} cm/s (the laboratory's procedure without precise control on compactive energy) and 2.1×10^{-9} cm/s (standard Proctor). These differences, too, were considered relatively minor.

3. The testing laboratory conducted the hydraulic conductivity tests in a flexible-wall permeameter with an effective stress of 103 kPa (15 psi). The soils were to be used in a final cover. The anticipated vertical compressive stress in a cover is much less than 103 kPa. An increase in confining stress produces a decrease in hydraulic conductivity. Thus, concern was expressed that the hydraulic conductivity tests at 103 kPa may have produced hydraulic conductivities that were too low for a final cover application involving lower confining stress. Additional test specimens were compacted and permeated at a confining stress that is more appropriate for a final cover system, i.e., 14 kPa (2 psi). Then the confining stress was increased to 103 kPa (15 psi), and the specimen, after consolidation, was repermeated. On the average, increasing the confining stress from 14 to 103 kPa decreased the hydraulic conductivity by a factor of 2.7. To account for this, all hydraulic conductivities measured at 103 kPa confining stress were multiplied by 2.7 to estimate the value at 14 kPa.

After all this was done, the data were submitted to the regulatory agency. The agency decided that its rules were hard and fast and would not deviate from them. Thus, the proposed acceptable zone shown in Fig. 5 was rejected. Naturally, the owner and author were disappointed.

The lessons learned from this case history can be summarized as follows:

1. The acceptable zone concept for hydraulic conductivity can be applied, but care needs to be taken to define compaction and testing criteria at the outset.

2. Deviation from regulatory requirements, no matter how logical from a technical perspective, will not always be met with approval from the regulatory agency.

Site in Southeast

This case history involved restoration of contaminated areas and construction of a clay cap consisting of 0.9 m (3 ft) of compacted clay that was required to have a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s. The clay barrier was overlain by a layer of protective soil.

The clay was kaolinite obtained from a commercially-operated mine located within 25 km of the site. Average properties of the clay were liquid limit = 69%, plasticity index = 37%, percent fines = 98%, optimum water content and maximum dry unit weight (ASTM D-698) = 26.8% and 93 pcf, respectively. Construction specifications were written based on the results of an extensive test pad program and required the kaolinite to be placed at a water content that was 2 to 4 percentage points wet of the optimum value determined from standard compaction (ASTM D-698). The required placement water content was significantly greater than the natural water content; water was added to the soil in a preprocessing area to bring the water content of the soil up to the required value. Later in the project, the water content specification was amended to allow occasional outliers from the range of 2% to 4% wet of optimum, but the average water content still had to be in this range.

At the time the test pads were constructed (1988), the average optimum water content (w_{opt}) was about 27%. Construction started in the summer of 1989. The optimum water content and maximum dry unit weight of the kaolinite during late 1989 were similar to values measured during the test pad program. However, starting on 30 January, 1990, the average optimum water content dropped several percentage points from 25.3% to 22.1%. Accordingly, soils were placed at lower water contents to conform to the construction specifications, which called for average water contents in the range of 2% to 4% wet of optimum. Although the optimum water content rose slightly in March, 1990, it was still 2 percentage points below the optimum value used initially despite the fact that the materials came from the same commercial kaolinite mine. In early April, the optimum water content rose back to a value close to the original value of approximately 27%. During the period January 30, 1990, to April 5, 1990, when lower placement water contents were observed, approximately 40,000 yd3 of kaolinite were placed and compacted.

In the summer of 1990, the author was asked to review the data and offer an opinion about (1) whether the drop in optimum was real or was the result of some change in testing procedures, and (2) what should be done with the material that had been placed at lower water contents than the other materials. Hydraulic conductivity tests on undisturbed samples of soil were not a part of the quality assurance program -- instead, the favorable results from hydraulic conductivity tests on large-scale test pads were used along with indirect quality control/quality assurance tests and observations to establish that the actual liner was built to standards that equal or exceeded those used in the test pad program. Thus, an assessment about whether or not the lower placement water contents for the period in question represented a possible problem had to be made based on other information besides hydraulic conductivity results.

At that time, the author reviewed available data, met with the engineers who were involved in the project, and attempted to develop an explanation for the drop in optimum water content. A review of soil data showed that the liquid limit of the soils also shifted during the period in question. A plot of liquid and plastic limit versus time is shown in Fig. 6. During the initial stages of construction, the liquid limit of the kaolinite was between 68% and 71%, which is practically identical to the average liquid limit of 69% measured during the test pad program. During the period of January to April, 1990, the liquid limit dropped to 60 to 66%, but then, beginning in April, 1990, the value rose and fell within the original range of 68 to 71%. The plastic limit also changed slightly (Fig. 6), but the shifts were very small (only 1 to 2%) and were not linked to changes in optimum water content.

The procedures used in compacting the soil were carefully examined. During the test pad program, a mechanically-operated drop hammer was employed to deliver the compactive energy, and later a manually-operated drop hammer was sometimes used. Comparative tests indicated

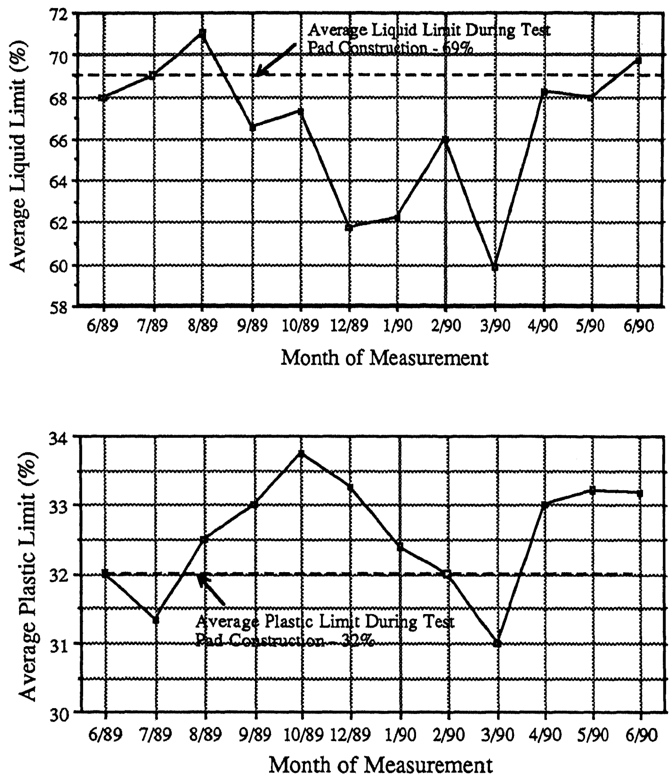


Figure 6. Change in Atterberg Limits with Time.

that the type of hammer drop mechanism (hand-operated or manual) had no effect. Further, because the liquid limit dropped significantly during the same time period in which the optimum water content dropped, it seemed clear that changes in compaction procedures alone could not explain the drop in optimum water content. It was assumed that the same mechanism caused both the decline in liquid limit and the decline in optimum water content -- changes in compaction procedure could not have affected liquid limit.

Other explanations for the changes in soil properties were sought. One possible explanation for the drop in liquid limit and optimum water content was that the borrow soil had changed. While this possibility could never be completely ruled out, changes in the soil itself seemed unlikely because (1) the soil was excavated from the same mine with relatively uniform, massive kaolin deposits, and (2) the plastic limit of the soil did not change significantly (if the material had changed, it seems probable that the plastic limit would have changed, too).

A statistical analysis of all the Atterberg limits and compaction data was performed to determine if the variations might be the result of random shifts in soil properties. The changes in optimum water content and liquid limit were much larger than the statistical noise in the data -- the trends were real and not just random variations.

A final hypothesis was that changes in soil preparation procedures had caused the drops in optimum water content and liquid limit. The degree to which the soil was dried during preprocessing of the material, and the subsequent period of rehydration with water, varied. During the period in question (January - April, 1990), and possibly slightly longer, the soil was apparently processed by drying the soil outdoors. There extent of drying was not documented, but it is thought that at least some soils were dried significantly. It is well known that drying of clayey soils lowers the liquid limit of the soil. However, no published data could be found on changes in optimum water content caused by air drying of clay soils, except for data on lateritic soils from tropical areas.

Based on these considerations, the author concluded that it is possible that the drop in optimum water content was due to a change in the soil, in which case a drop in placement water contents was appropriate. However, an equally (and probably more) probable explanation for the drop in optimum is that the soil was dried to a greater extent during the period in question, and that the drop in optimum was a result of changes in soil preparation procedure. If this explanation is correct, then soils were placed at too low a water content.

The consequence of placing the soil at too low a water content was considered. Unfortunately, an acceptable zone had not been established with the procedures discussed earlier. While it was known from the test pad program that compaction of the soil to the project specifications produced a satisfactory material, it was not known how much deviation from the specification would still lead to acceptable results. Based on limited data from the test pad, the author constructed a conservative acceptable zone (Fig. 7). This acceptable zone was applied to the field-constructed liner, and a recommendation was made to replace areas that fell outside this zone.

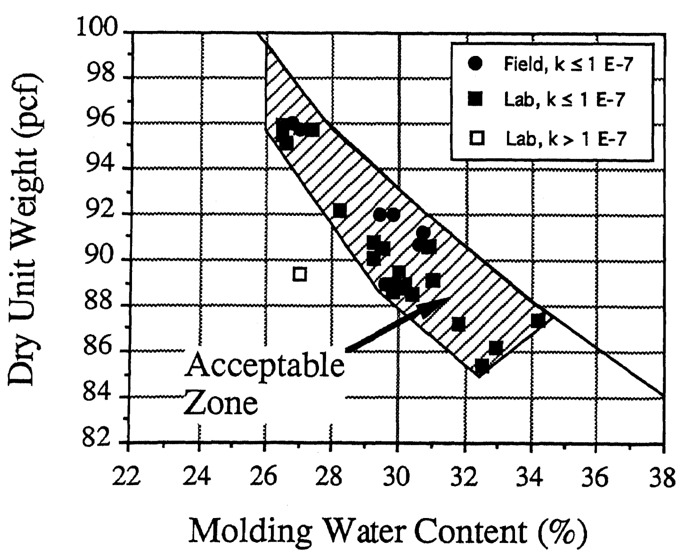


Figure 7. Acceptable Zone Based on Available Data.

Some months later, the authors obtained kaolinite fron same borrow pit and performed tests to study the effect (predrying on the Atterberg limits and optimum water coı of clayey soils. Results are described by Chao (1991). (found (Fig. 8) that air drying did cause a reduction in optimum water content and an increase in dry unit weigł

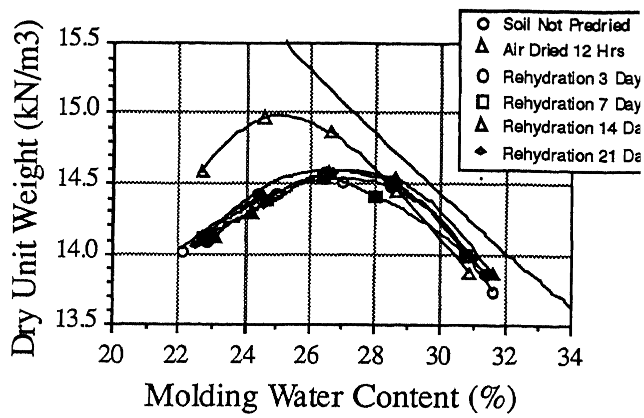


Figure 8. Shift in Compaction Curves.

The standard and modified compaction procedures (ASTM D698 and D1557, respectively) specify that a representative portion of soil be sieved (e.g., through th(4 sieve) prior to compaction. The standards state:

> "If the sample is too damp to be friable, reduce the moisture content by drying until the material is friable ... Drying may be in air or by the use of a drying apparatus such that the temperature does not exceed 140°F (60°C)."

The standards require that:

> "Whenever practicable, soils classified as ML, CL, OL, GC, SC, MH, CH, OH, and PT by Test Method D2487 shall be prepared in accordance with 4.1.4."

Section 4.1.4 is the "Moist Preparation Procedure." For moist preparation, the soil is processed "...without previously drying the sample." Section 4.1.4 states that ı moist preparation procedure is "recommended" for the s(classifications listed above. The fact that moist preparati is recommended, but not required, gives the operator lati in the degree of drying to which the soil is subjected. Th lack of precise control over predrying of the soil is not required by the ASTM standard (but probably should be required).

ı summary, the following lessons were learned from this ꞓ history:

The amount of air drying of a soil can affect the compaction characteristics.

If the amount of predrying changes during a project, the compaction characteristics can change, which will cause changes in placement water contents that may not be appropriate.

Engineers should pay close attention to sample preparation procedures and guard against changes in those procedures -- ASTM procedures do not guarantee that there will not be a significant change that can affect the results of compaction tests.

Definition of an acceptable zone of water content and dry unit weight values can be valuable in helping to define how large a deviation from construction specifications is acceptable, which can be helpful if problems develop during construction.

SULTS OF LARGE-SCALE HYDRAULIC NDUCTIVITY TESTS ON TEST PADS

est pads are constructed to verify that the materials and hods of construction proposed for a project will produce lesired results, i.e., low hydraulic conductivity. raulic conductivity is generally verified with a large- ꞓ field test, such as the sealed double-ring infiltrometer, DRI (Daniel, 1989a). The SDRI (Fig. 9) consists of two s. The square outer ring typically measures 3.6 m by n (12 ft by 12 ft), and the square inner ring is 1.5 m (5 ide on all sides. The outer ring is open, but the top of nner ring is sealed. The rings are imbedded into onite-backfilled trenches, filled with water, and itored. A flexible bag is attached to the inner ring, ch is periodically removed and weighed to determine the r flux. Tensiometers are used to monitor the depth of vetting front. Hydraulic conductivity is computed from nown flux of water and estimated hydraulic gradient.

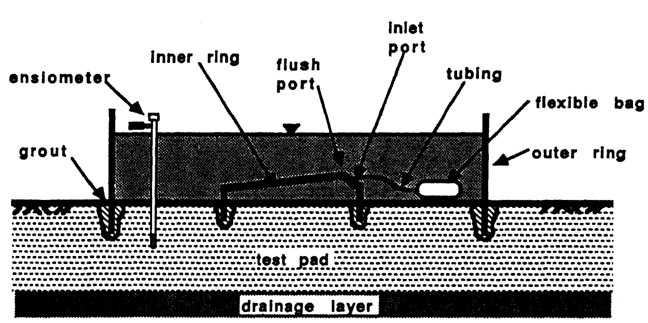


Figure 9. Sealed Double Ring Infiltrometer.

Performance of Eight Test Pads in Texas

Mikus (1989) collected and analyzed all of the data available at the time on results of test pads constructed in Texas and presented in support of permit application for hazardous waste treatment, storage, or disposal facilities. The following discussion is based on data from Mikus (1989).

Table 3 summarizes data on the soils and the properties of the soils from the various test pads. The thickness, type of compactor, and hydraulic conductivity measured with the SDRI are reported, also. In some cases, more than one SDRI was used.

Some of the more interesting findings from these experiences are the following:

- Transwestern Pipeline Company, Fort Stockton, Texas. This test pad measured 60 m (200 ft) by 30 m (100 ft) in plan dimensions and was divided into 8 cells. The water content of the soils, number of passes of the roller, and other construction parameters were varied from cell to cell. Based upon practical experience and quality control test data, one of the cells was selected for testing. Very successful results were obtained (hydraulic conductivity was 1×10^{-8} cm/s).

- Phillips 66 Company, Borger, Texas. Two test fills were constructed, with the amount of bentonite added to native, lean clays varied from 6.5% at Test Pad 1 to 8.5% at Test Pad 2. A pulverizer was used to mix the native soil with bentonite in either 3 passes (for the 6.5% bentonite mixture) or 2 passes (for the 8.5% bentonite mixture). The processed material was placed in 8-in. thick loose lifts and compacted with a minimum of 6 passes of the roller. The test pad results are contrasted in the table below:

Parameter	Test Pad with 6.5% Bentonite	Test Pad with 8.5% Bentonite
Optimum Water Content (ASTM D-698)	17.2% - 19.9%	18.8% - 21.5%
Average Placement Water Content	18.4%	21.0%
Maximum Dry Unit Weight (ASTM D-698)	105.8 - 108.7 pcf	104.8 - 105.3 pcf
Percent Compaction of Test Pad	96	96 - 98
Liquid Limit of Soil-Bentonite Mixture	56	65
Plasticity Index of Soil-Bentonite Mixture	31	39
Hydraulic Conductivity Measured with SDRI	2×10^{-8} cm/s	1×10^{-7} cm/s

Table 3. Summary of Data from Test Pads in Texas (from Mikus, 1989).

Site	Description of Soil	Liquid Limit (%)	Plasticity Index (%)	Percent Passing No. 200 Sieve	Percent Compaction of Soil	Type of Roller	Lift Thickness &Total Thickness	Hydraulic Conductivity (cm/s)
Celanese Engrg. Resins (Bishop, Texas)	Native Silty Clay	66	42	76	95 - 104 (ASTM D-698)	Wedge Foot	6-in. Lifts, 2-ft Thick	4×10^{-8}
Transwestern Pipeline (Fort Stockton, Texas)	Native Clay	38	19	99	97 - 105 (ASTM D-698)	Vibratory Padfoot (Vibration Not Used)	6-in. Lifts, 2.5-ft Thick	1×10^{-8}
Phillips 66 Company (Borger, Texas) [Test Fill 1]	6.5% Bentonite Mixed with a Red, Lean, Native Clay	56	31	55	96 (ASTM D-698)	CAT 815B	6-in. Lifts, 3-ft Thick	2×10^{-8}
Phillips 66 Company (Borger, Texas) [Test Fill 2]	8.5% Bentonite Mixed with a Red, Lean, Native Clay	65	39	63	96 - 98 (ASTM D-698)	CAT 815B	6-in. Lifts, 3-ft Thick	1×10^{-7}
Union Carbide (Seadrift, Texas) [Test Fill 1]	Onsite Brown and Yellow Clay with Silt Pockets	60 - 75	42 - 47	Not Given	Not Given	CAT 815B	4-in. Lifts, 2-ft Thick	5×10^{-8}
Union Carbide (Seadrift, Texas) [Test Fill 2]	Offsite Light Brown Sandy Clay	35 - 55	23 - 35	Not Given	Not Given	CAT815B	4-in. Lifts, 2-ft Thick	2×10^{-8}
Dupont (Victoria, Texas) [Test Fill 1]	Dark Gray Clay	63	42	80	Not Given	CAT 815B	6-in. Lifts, 3.5-ft Thick	3×10^{-8} 4×10^{-8}
Dupont (Victoria, Texas) [Test Fill 2]	Tan Clay	52	35	80	Not Given	CAT 815B	6-in. Lifts, 3.5-ft Thick	4×10^{-8} 3×10^{-8}
Shell Oil Company (Deer Park, Texas)	Tan Sandy Clay	30 - 50	15 - 37	53 - 82	95 - 104 (ASTM D-698)	CAT 815B	1- to 5-in. Lifts, 2.5-ft Thick	4×10^{-8} 5×10^{-8}
BP Chemicals America (Port Lavaca, Texas) [Std. Proctor Section]	Sandy, Silty Clay	28 - 62	12 - 41	53 - 97	96 - 100 (ASTM D-698)	CAT 815	6-in. Lifts, 2-ft Thick	1×10^{-7} 8×10^{-8}
BP Chemicals America (Port Lavaca, Texas) [Mod. Proctor Sect.]	Sandy, Silty Clay	28 - 62	12 - 41	53 - 97	91 - 93 (ASTM D-1557)	CAT 815	6-in. Lifts, 2-ft Thick	2×10^{-7}
Gulf Coast Waste Disp. (Texas City, Texas) [Test Area A]	Reddish Brown or Tan Clay	47-74	39 - 46	70 - 90	98 - 99 (ASTM D-698)	Vibratory Padfoot (Vibration Not Used)	6-in. Lifts, 2-ft Thick	2×10^{-7}
Gulf Coast Waste Disp. (Texas City, Texas) [Test Area B]	Reddish Brown or Tan Clay	47-74	39 - 46	70 - 90	94 - 98 (ASTM D-698)	Vibratory Padfoot (Vibration Not Used)	6-in. Lifts, 2-ft Thick	4×10^{-7}

Strangely, the addition of additional bentonite did not cause a reduction in hydraulic conductivity -- curiously, it caused an increase. An explanation for the counter-intuitive result is not apparent.

- <u>BP Chemicals America, Inc., Port Lavaca, Texas</u>. The test pad at this facility was divided into two sections. In one section, compaction was controlled using the results of standard Proctor compaction (ASTM D-698), with the requirement that the dry unit weight be no less than 95% of the maximum value from ASTM D-698. In the other section, compaction was controlled using the results of modified Proctor compaction (ASTM D-1557), with the requirement that the dry unit weight be no less than 90% of the maximum value from ASTM D-1557. For the "standard section," optimum water content and maximum dry unit weight from ASTM D-698 were 20.4 to 23.6% and 96 to 100 pcf, respectively. The soil was compacted 1.4 to 4.6 percentage points wet of optimum to a percent compaction of 96 to 100%. For the "modified section," optimum water content and maximum dry unit weight from ASTM D-1557 were 16.0 to 18.3% and 111 to 114 pcf, respectively. The soil was compacted 2.0 to 4.3 percentage points wet of optimum to a percent compaction of 91 to 93%. The average hydraulic conductivities were as follows:

  - Standard Section: 9×10^{-8} cm/s
  - Modified Section: 2×10^{-7} cm/s

In one case (with standard Proctor control) the test pad met the hydraulic conductivity requirement of 1 x 10^{-7} cm/s or less, but with modified Proctor control, the test pad did not meet regulatory criteria. The placement water contents and dry unit weights compare as follows:

Test Section	Placement Water Content (%)	As-Compacted Dry Unit Weight (pcf)
Standard	23 - 27	100 - 104
Modified	19 - 22	102 - 104

It appears that the soils were compacted to similar dry unit weights in the two sections but to significantly higher water contents in the "standard" test section. Experience in general on the Texas Gulf coast has been that the highly plastic clays of the region have to be wetted to relatively large water contents in order to achieve satisfactory results.

- Gulf Coast Waste Disposal Authority, Texas City, Texas. One test pad was constructed, but the test pad was divided into two sections. The compactive energy was varied from one section to another. The first lift of soil in the test pad was compacted with 10 and 6 passes for the high and low compactive energies, respectively, and subsequent lifts were compacted with 16 and 8 passes, respectively. An Ingersoll Rand, self-propelled, vibratory padfoot roller was employed for compaction, but vibration was not used. In this case, the high compactive energy yielded a compacted soil having a hydraulic conductivity of 2 x 10^{-7} cm/s, whereas the lower energy produced material with a hydraulic conductivity of 4 x 10^{-7} cm/s. Increasing compactive energy did produce benefit.

The lessons learned from these case histories may be summarized as follows:

- Dividing a large test pad into cells and experimenting with construction variables (e.g., water content of the soil and number of passes) can be very helpful in determining the optimum construction procedure to use for a compacted soil liner.

- The addition of more bentonite to a native soil does not always cause a reduction in hydraulic conductivity.

- For some soils (especially highly plastic materials), placement water content is critical. Using standard Proctor for control tends to force a higher placement water content than modified Proctor, and for this reason, control of construction with standard Proctor compaction can produce more desirable results for soils that must be moistened to relatively high water contents in order to achieve low hydraulic conductivity.

- Increasing the number of passes of a roller can produce a measurable reduction in hydraulic conductivity.

Collection of Field Hydraulic Conductivity Measurements

In the spring of 1990, the U.S. EPA was developing regulations that govern disposal of non-hazardous solid waste under Subtitle D of the Resource Conservation and Recovery Act, or RCRA. The author was asked to consider available information concerning the thickness of compacted soil liners and to make a recommendation concerning a reasonable minimum thickness for soil liners in municipal solid waste landfills. At the time, the thickness of liners varied from 0.3 to 1.5 m (1 to 5 ft), depending upon the local regulatory agency's rules.

A data base on hydraulic conductivity was assembled. The data base consisted of case histories of large-scale field measurements of hydraulic conductivity. The data base was restricted to in situ measurements because laboratory measurements can sometimes yield unrepresentative values (Daniel, 1984; Day and Daniel, 1985; and Elsbury et al., 1990). Most of the data come from large-scale hydraulic conductivity tests on tests, but many data points come from pan lysimeters located beneath liners.

A summary of the data is presented in Table 4. For each value of in situ hydraulic conductivity, a description of the quality of construction is provided. When construction practices were undocumented, the liner was assumed to have been built by poor standards. Soils compacted with modest-sized compaction equipment, or those compacted with somewhat questionable means (such as compacting the soil dry of optimum) are generously rated as "good" quality construction provided full-sized equipment was used and documentation was extensive. "Excellent" quality of construction is used to describe construction in the field with heavy equipment and generally good construction practices; data documentation was thorough. No small-test cells constructed with hand-held tampers were included in the data set; only soils compacted in the field with self-propelled or towed rollers were considered.

The hydraulic conductivities are plotted as a function of thickness of the liner in Figure 10. All the data in Table 4 are included in this plot. Figure 11 presents the same type of information, but just for soils for which the quality of construction was judged to be "good" or "excellent."

Figures 10 and 11 show a tendency for in situ hydraulic conductivity to decrease with increasing thickness of the liner. Sensitivity to the thickness of a liner is most pronounced for liners < 0.6 m (2 ft) in thickness. For soil liners at least 0.6-m (2-ft) thick, there is only a small decrease in hydraulic conductivity with increasing thickness. On the average, a large reduction (an order of magnitude) in hydraulic conductivity occurs when the thickness of the liner is increased from 0.3 m (1.0) to 0.45 m (1.5 ft). With each succeeding 0.15 m (0.5 ft) increase in thickness, the average hydraulic conductivity is approximately halved.

Table 4. Summary of Large-Scale Field Measurements of Hydraulic Conductivity of Soil Liners.

Reference	Description of Site	Plasticity Index (%)	Quality of Construction	Thickness (ft)	Method of Hydraulic Conductivity Measurement	Hydraulic Conductivity (cm/s)
Daniel (1984)	Central Texas	20	Unknown	1.0	Leak Rate	4×10^{-5}
	Northern Texas	--	Unknown	0.7	Infiltrometer	3×10^{-6}
	Southern Texas	23 - 55	Unknown	2.0	Leak Rate	2×10^{-5}
	Mexico	14 - 24	Good	1.6	Leak Rate	1×10^{-6}
Day & Daniel (1985)	Prototype 1	11	Poor	0.5	Underdrain	9×10^{-6}
	Prototype 2	45	Poor	0.5	Underdrain	4×10^{-6}
Rogowski (1986)	Test Pad	12	Good	1.0	Underdrain	5×10^{-7}
Daniel and Trautwein (1986)	Cover	--	Excellent	3.0	SDRI	8×10^{-8}
Daniel (1987)	Confidential	--	Excellent	1.0	Leak Rate	2×10^{-6}
Lahti et al. (1987)	Keele Valley	7 - 15	Excellent	3.9	Lysimeter	9×10^{-9}
Goldman et al. (1988)	Site K	49 - 69	Good	1.0	Lysimeter	1×10^{-7}
Gordon et al. (1989)	Marathon Coun	16-54	Excellent	4.0	Lysimeter	2×10^{-8}
	Marathon Coun.	16-54	Excellent	4.0	Lysimeter	5×10^{-9}
	Portage County	13-33	Excellent	5.0	Lysimeter	5×10^{-9}
	Sauk County	13-33	Excellent	5.0	Lysimeter	2×10^{-8}
Albrecht and Cartwright (1989)	Test Pad	7	Excellent	3.0	SDRI	4×10^{-8}
Mikus (1989)	Celanese	42	Excellent	2.0	SDRI	4×10^{-8}
	Transwestern	19	Excellent	2.5	SDRI	1×10^{-8}
	Phillips 66	31	Excellent	3.0	SDRI	2×10^{-8}
	Phillips 66	39	Excellent	3.0	SDRI	1×10^{-7}
	Union Carbide	42 - 47	Excellent	2.0	SDRI	5×10^{-8}
	Union Carbide	23 - 35	Excellent	2.0	SDRI	2×10^{-8}
	DuPont	42	Excellent	3.5	SDRI	3×10^{-8}
	Du Pont	35	Excellent	3.5	SDRI	3×10^{-8}
	Shell Oil Co.	15 - 37	Excellent	2.5	SDRI	4×10^{-8}
	BP Chemicals	12 - 41	Excellent	2.0	SDRI	9×10^{-8}
	BP Chemicals	12 - 41	Excellent	2.0	SDRI	2×10^{-7}
	Gulf Coast Waste	39 - 46	Excellent	2.0	SDRI	2×10^{-7}
	Gulf Coast Waste	39 - 46	Excellent	2.0	SDRI	4×10^{-7}
Krapac et al. (1989)	Test Pad	10	Excellent	3.0	Infiltrometers	4×10^{-8}
Elsbury et al. (1989)	Test Pad	41	Poor	1.0	Underdrain	1×10^{-4}
Clough-Harbour (1989)	Test Pad	--	Good	2.0	SDRI	1×10^{-7}
Fernuik and Haug (1990)	Residual Soil	11 - 14	Excellent	2.0	Infiltrometer	2×10^{-7}
Johnson et al. (1990)	Liner A	35	Excellent	2.0	SDRI	3×10^{-8}
	Liner B	34	Excellent	2.0	SDRI	1×10^{-8}

There are 23 soil liners listed in Table 4 that were built with good to excellent construction practices and that had thicknesses of 0.6 m (2.0 ft) or more. Of these 23 liners, 22 (or 96%) had hydraulic conductivities $\leq 1 \times 10^{-7}$ cm/s. The one soil liner that did not have an in situ hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s had a value of 2×10^{-7} cm/s, and this value is debatable; with longer-term testing, the in situ hydraulic conductivity might very well have been found to be $\leq 1 \times 10^{-7}$ cm/s.

The case histories summarized in Table 4 represent a wide range of construction practices. It is certainly possible to expect that in the data base, thick liners may be constructed more carefully than thin liners. Because the case histories represent available case histories, and not necessarily a representative sampling of field cases, there may be a bias in the data base. Nevertheless, the results shown in Figs. 10 and 11 do seem reasonable and consistent with what one might expect. It is assumed that each lift of soil contains a few hydraulic defects (zones of inadequate compaction, zones of poorer materials, minute cracks, etc.). The more lifts there are, the lower is the probability of a continuous path of flow developing through the interconnected defects. Thus, it seems that liners with only 1, 2, or 3 lifts have a higher probability of performing poorly than liners with more lifts.

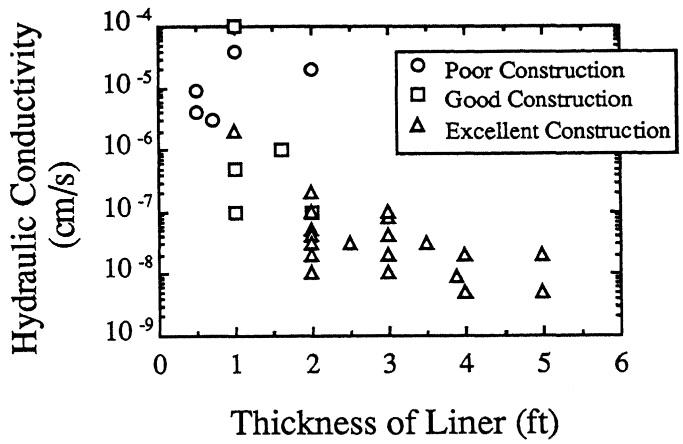


Figure 10. Field Hydraulic Conductivity Versus Thickness of Liner (Data from Table 4).

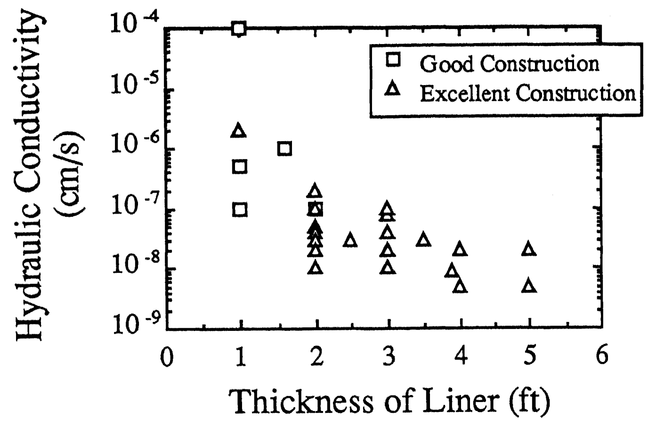


Figure 11. Field Hydraulic Conductivity Versus Thickness of Liner for Soil Liners Constructed with Good or Excellent Procedures (Data from Table 4).

Based on these data, the following conclusions were drawn:

1. With sound construction practices, one should be able to construct a soil liner that has an in situ hydraulic conductivity that is less than or equal to 1 x 10^{-7} cm/s if the soil liner is at least 0.6 m (2.0 ft) thick.

2. The data base presented indicates that soil liners with a thickness of less than 0.6 m (2.0 ft) have an high probability of having an in situ hydraulic conductivity greater than 1 x 10^{-7} cm/s. For this reason, soil liners thinner than 0.6 m (2.0 ft) are not recommended unless data are developed for the proposed materials and construction practices to demonstrate that the liner will have an adequately low in situ hydraulic conductivity at thicknesses less than 2 ft.

The EPA considered this information, along with other information, and decided to require a minimum thickness of 0.6 m (2 ft) for the compacted clay liner component of a geomembrane/clay composite liners for municipal solid waste landfills. The main lesson learned from this exercise was that one can use a data base of large-scale, field hydraulic conductivity tests to draw important conclusions about how certain variables (such as the thickness of a liner) may influence the performance of the liner.

CASE HISTORIES INVOLVING FINAL COVERS

Final cover systems for waste disposal facilities offer unique challenges for a compacted clay liner. Some of the special requirements include:

- The soil is particularly sensitive to construction defects because the soil liner is subjected to a very low overburden stress -- bottom liners for landfills are not so sensitive because the compressive stress from overlying materials will tend to close any macropores and to reduce hydraulic conductivity (Daniel, 1987).

- The soil is more vulnerable to damage from freeze-thaw, which is known to damage clay soils (Othman and Benson, 1991; Kim and Daniel, 1992; and references therein).

- The soil is more vulnerable to damage from desiccation, which can crack the clay liner.

- The soil is vulnerable to damage from differential settlement of the underlying waste -- differential settlement can produce tension in the compacted clay, which can crack and damage the clay (Gilbert and Murphy, 1987; and Jessberger and Stone, 1991; and Scherbeck et al., 1991)

These problems make the design of a compacted clay within a final cover a significant challenge in many cases.

Final Cover over Pulp and Paper Mill Waste

This particular project involved placement of a final cover system on 3H:1V slopes on a mound, approximately 30 m (100 ft) high, of organic waste material from a pulp and paper mill. Although the waste was "dewatered" before being placed in the waste heap, the waste material was rich in organic matter and contained significant moisture. The waste heap had been built up over a period of many years at a site near the Canada-U.S. border.

The final cover system consisted, as shown in Fig. 12, of a 0.6-m (2-ft) thick layer of compacted clay (hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s) overlain by 6 inches of protective soil. Gas vents were provided at a frequency of roughly 1 per acre. The vents were connected with shallow gravel-filled trenches -- there was not a continuous blanket of granular material beneath the clay.

About two months after construction of the cap was completed, cracks began to develop in the cover. The cracks ran along the slope at a more-or-less constant elevation, but at different elevations up and down the slope. The cracks appeared were as much as 150 mm (6 in.) wide and extended in some cases through the cover soil and compacted clay. There was no evidence of vertical offset at the cracks and no evidence of any deep-seated failure surface beginning to develop. It was obvious that the foundation was not very stable, that the cover material was undergoing large deformation, and that the clay ceased to have a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s.

This case history provides an excellent example of the challenges faced by designers of final cover systems. In the author's opinion, the design was a faulty one for several reasons:

1. The depth of cover soil (6 in.) was inadequate to protect the underlying compacted clay liner from damage due to freezing temperatures at this site near the U.S.-Canada border.

2. The depth of cover soil was inadequate to protect the underlying compacted clay liner from damage due to desiccation during prolonged dry periods.

3. The cover soil underwent significant erosional losses -- gullies had fully penetrated the thin cover soil in just a few months.

4. The large differential settlement would eventually cause severe cracking in the compacted clay liner.

5. The wet, gas-producing waste was not vented as well as it could have been -- build up of gas pressure probably weakened the waste and contributed substantially to the severe cracking in the cover.

It is the author's opinion that a compacted clay liner designed for a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s should not be used in a final cover system unless the liner is protected from freezing and desiccation damage. Further, because differential settlement can crack compacted clay liners, a compacted clay liner designed to maintain a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s for a long time should not be constructed on top of unstable waste that will cause large differential settlement. An alternative for waste that will undergo large settlement is a thin geosynthetic clay liner, which appears to have much greater capability for withstanding large differential settlement (LaGatta, 1992).

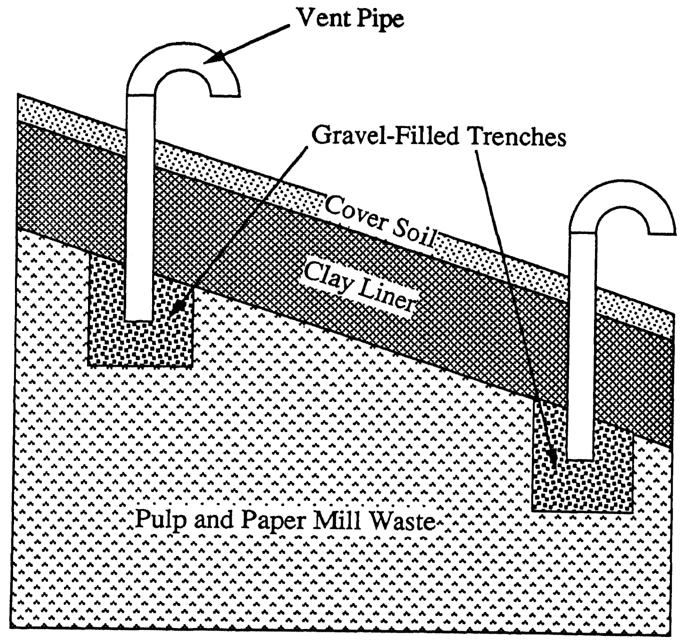


Figure 12. Cover System for Site Near U.S.-Canada Border.

Final Cover Over Superfund Site

This case history involved placement of a final cover system over a 60-acre landfill that contains municipal solid waste and hazardous waste. Ground water at the site is being extracted and treated -- a final cover was needed to minimize infiltration of water into the waste and thereby to minimize further generation of leachate.

The original design is shown in Figure 13a. The design called for a 0.6-m (2-ft) thick layer of compacted clay overlaid by 2 ft of protective topsoil. The design was originated in the mid 1980's, and at the time, this type of design was common. However, in the past several years, designers have come to recognize the value of using composite geomembrane/clay liners, which are expected to be far less permeable than either geomembrane liners alone or clay liners alone.

As this remediation project approached the construction stage in the early 1990's, the design/construction team reconsidered the alternatives. Among the problems with utilization of compacted clay at this site was the fact that no clay soils were available locally -- the clay would have to be trucked a distance of about 25 km, which would have caused considerable disruption to several small communities. It was recognized that a geomembrane would offer nearly complete resistance to infiltration of water, but the geomembrane may have occasional defects, particularly in seams, and it was desired to have clay beneath the geomembrane.

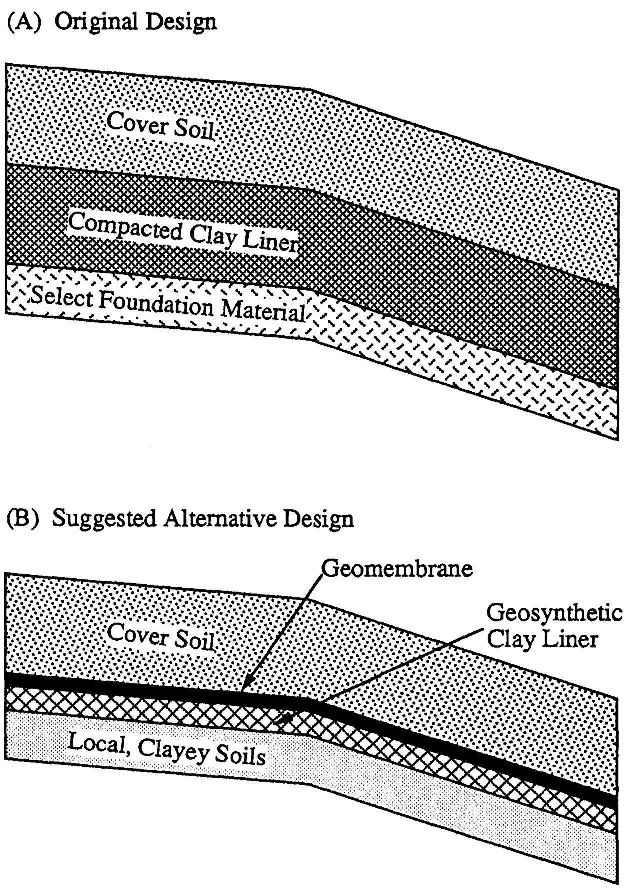


Figure 13. Cover Designs for Superfund Site.

An alternative design (Fig. 13b) was proposed. The barrier layer consisted of a composite liner containing a geomembrane and a geosynthetic clay liner (GCL). The GCL consists of a thin layer of bentonite either sandwiched between two geotextiles or glued to a geomembrane, depending on the specific product. Information on GCLs is provided by Daniel (1991b). Because of concern over the long-term shear strength of GCLs, the designers proposed to use compacted clay rather than GCLs on the relatively steeply-sloping edges of the landfill cover and to use the GCL in the relatively flat interior (which was most of the area of the cover).

The main problem, once it was decided that the proposed alternative was technically superior to the original cover, was convincing officials of the various regulatory agencies involved that the proposed alternative was acceptable. Concern was expressed that there was too little field experience with GCLs. One regulatory official told the author, "I'm all for innovative technologies, but not on my project." The process was essentially one of education in which regulatory officials who were not familiar with GCLs were given detailed briefings and written documentation about the properties of GCLs. Finally, the regulatory officials agreed to the alternative design.

The main lesson learned from this case history was that an alternative design to a conventional compacted clay liner in a final cover system may make sense technically, but convincing regulatory officials to approve the alternative design may require an extensive effort to educate and inform the officials about a new material or new approach.

CONCLUSIONS

In this paper, a number of case histories involving compacted clay liners have been described. Each case history had its own unique circumstances and lessons to be learned. However, lessons that seemed to be repeatedly relearned by the parties involved were: (1) compaction control is critical, and it is important to spend the time and money before construction to define appropriate compaction criteria; (2) soil preparation procedures for testing compacted clay liners in the laboratory are not explicitly defined in testing standards, and yet variations in preparation procedures can produce a large change in results; (3) test pads can provide valuable lessons on compaction requirements and can help to avoid costly mistakes later; and (4) final cover systems offer unique challenges in terms of the performance of compacted clay liners -- alternative designs, e.g., incorporating geosynthetic clay liners, may prove to be very attractive for many cover systems.

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